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PLATE GIRDER BRIDGES IN RAILWAY CONSTRUCTION

PART I.

Prevalence Due to Advantages Over Other Designs —Types of Plate Girder Spans in Use—Dimensioning and Stress Calculation—Variety of Loads

By C. H. MARRS, C.E.,

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The builders of iron and steel bridges for more than sixty years have maintained that steel bridges properly designed, with good material, efficiently maintained, and used according to the intention of their designers, have in every case proved entirely satisfactory and capable of indefinite endurance and that any cases of disaster have been due to some defect in the construction, to derailment, or other accidental occurrences which bring upon certain members unforeseen strains which they were not intended to carry.

Of all types of steel railway bridges the plate girder span is the most common, as 75 % of the extensive mileage of bridges built, is in the form of plate girder spans, and it is this great commercial demand which has suggested to the writer the importance of this subject.

Canada is still in her infancy in railroad construction, and in the future it will be necessary for a great many of our young engineers to have charge of the design and fabrication of plate girder bridges. Shop methods and erection facilities have so changed that text books do not give the young engineers all the information they require. It is not the object of this series of articles to explain the fundamental theories of plate girders, but to treat the subject from the standpoint of an engineer in actual practice, and to give an intelligible explanation of various snags which are not covered in text books, and which are either omitted in different specifications, or upon which they do not agree.

The advantages of plate girder construction over other forms of bridges are so evident that it is used wherever conditions will permit. One of these advantages is its solidity, by reason of which it is better fitted to resist injury from derailed trains, to prevent corrosion from sulphur gases, and to withstand the severe conditions of exposure which pertain to railway bridges. Another reason for the superiority of this form of bridge is its simplicity and uniformity of construction, there being a minimum likelihood for error in design. Moreover, the drawng-office expense and shop costs are reduced, as it is usually possible to standardize spans. The erection is completed in a shorter time than in the case of any other form of construction. There is usually more weight of material required in plate girder bridges than in lattice or truss spans, but this objection is more than counterbalanced by its cheaper pound price, the greater speed in the completion of the work and the longer durability in actual service.

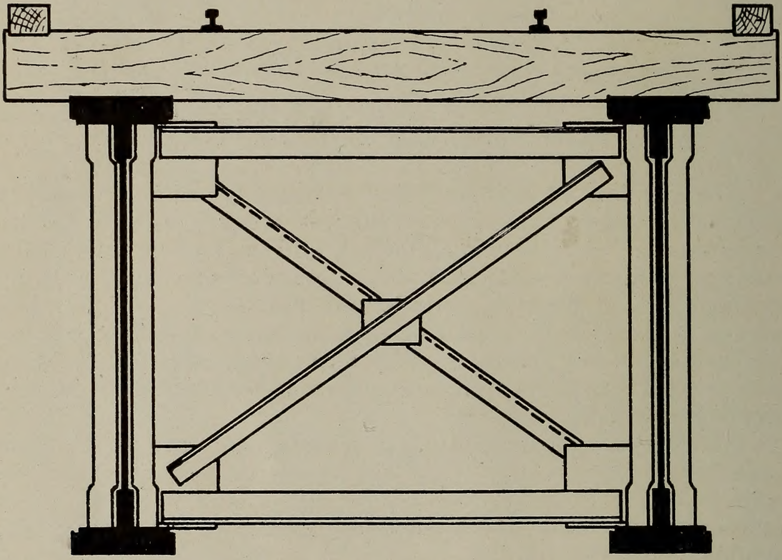


Fig. 1.—Type of Deck Plate Girder.

The design of a bridge may be divided into two branches; first, the determination of the type of bridge most suitable and its dimensions, and second, the calculation of stresses and sections of material. After the site of the proposed bridge has been surveyed and the engineer has determined the grade alignment of the track, the allowable depth of clearance line below the base of rail and the most suitable lengths of spans required, the design of the bridge can be attained.

Types.—The various types of plate girder bridges which are in general use at present are represented in Figs. 1, 2 and 3, the deck girder being the cheapest and simplest form and therefore used wherever conditions of clearance will permit. Where the allowable depth for clearance is reduced to the limit where a deck span would not suit, it is necessary to adopt the half deck girder, as shown in Fig. 2, or the through plate girder type, as shown in Fig. 3. The half deck girder span is very little heavier than the deck span and is frequently used where the top flange is not more than three feet above the base of rail, the main girders being spaced 13 feet between centres. There have been bridges built where the shelf angles have been lower than this, and in some structures the ties have been made to rest directly on the bottom flange. This is not good practice, as it is necessary to space the girders further apart to suit the recognized requirements of clearance, and this would demand longer and heavier ties and, moreover, the cross brace frames become so shallow that it is not possible to properly stay the top flanges of the main girders. The greatest objection to the half deck girder is that the ties resting on shelf angles must produce eccentric loading on the main girders, and this is not easily taken care of. The half deck span also requires extra heavy ties with an odd length, and these ties demand the most rigid inspection on account of the severe strain they undergo, and when it is necessary to replace them the procedure is much more difficult than in the case of the other types of girders because the ends of the ties butt into the webs of the girders.

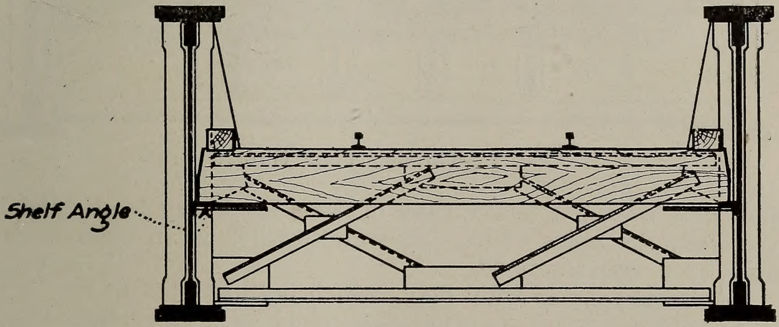


Fig. 2.—Half Deck Plate Girder.

The through plate girder span with stringers and floor beams, is undoubtedly the most desirable construction, where the depth of clearance is limited. Fig. 3 is an example of this form of bridge and shows four lines of stringers per track, which arrangement is considered to be better suited to support derailed trains, and which requires only a very light tie. But it is a cheaper construction, and quite as satisfactory, to use two lines of stringers per track.

In the case of railway bridges passing over streets in cities, where very shallow construction is an advantage, and where protection is needed for the street traffic, a solid floor is used. This may be accomplished by I-beams with a continuous cover plate, or by trough floor construction covered with a waterproofed concrete slab and ballast. The best type of trough floor is shown in Fig. 4. It is an expensive bridge and is not in very common use in Canada, but as railroads multiply in the larger cities it will be more necessary. The trough floor can be made shallower than any other type of floor for heavy loads, and is free from the noise of passing trains, which is an objection in more open construction.

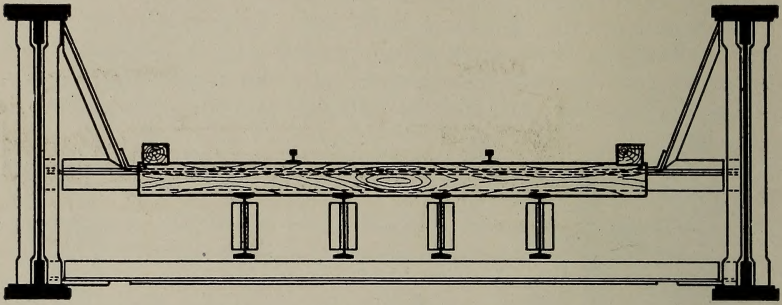


Fig. 3.—Through Plate Girder.

Another type of solid floor, which is illustrated in Fig. 5, is being used in Canada by several railroads, even on bridges where protection below is not needed. It consists of regular track ties in ballast supported on concrete, which covers and fills in between the cross I-beams over the main girders. This construction is being used because of the scarcity of good large bridge ties, and the expense which is frequently required to place these ties. Any construction which decreases the demand for timber is to be recommended, and there is a probability that bridge design may be considerably changed to suit the increasing scarcity of bridge ties.

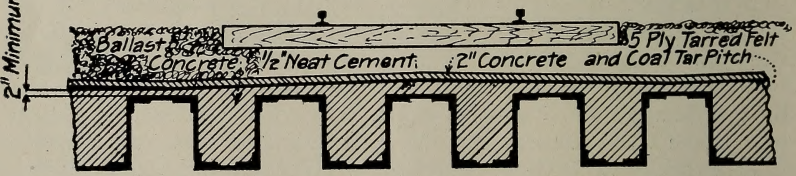


Fig. 4.—Cross Section Through Trough Floor.

There are some tube bridges still in existence, but they are not recommended in modern construction; nor is any form of box girder desirable where the inner surface is not accessible for painting. Wherever a box girder is used, it should be properly braced with diaphragms, so that it will not change its rectangular shape.

Dimensions.—In single spans the effective length of girders is dependent on the adopted clear opening between the abutments, as the edge of the masonry bearing plate should be six inches back from the face of masonry under coping. The length of the bearing plate can then be decided by an approximate calculation of the maximum end reaction. If the final calculations should give a slightly different required area of bearing plate, the difference can usually be taken care of in its width. The effective length of girders is the distance between the centres of these end bearing plates.

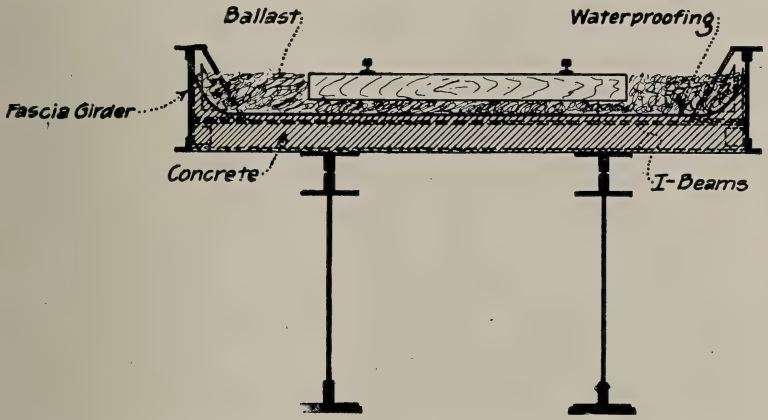
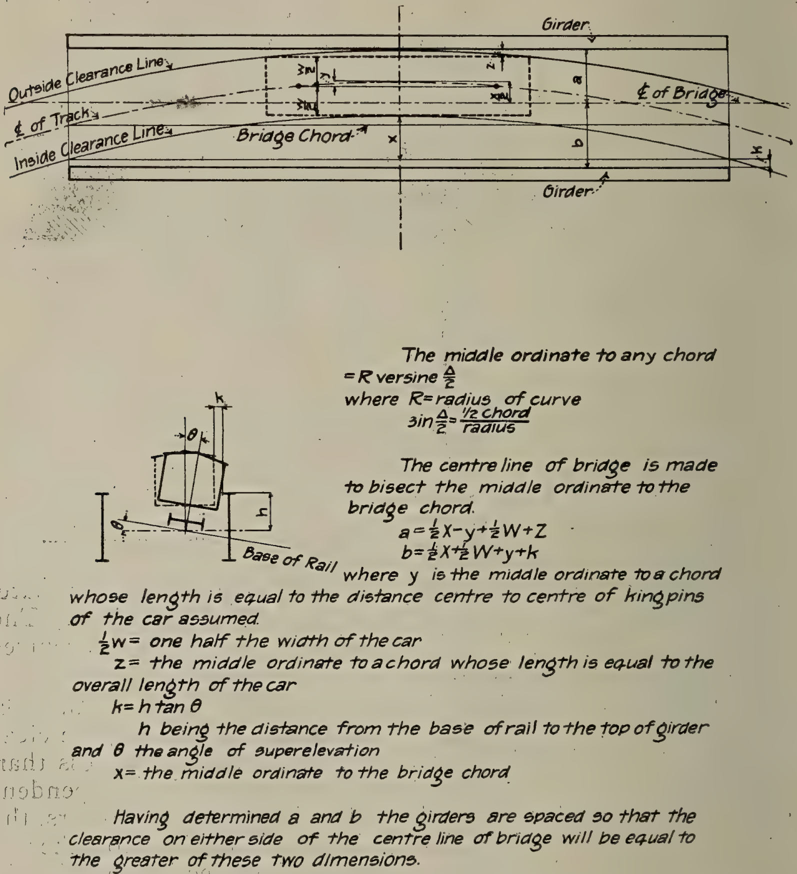


Fig. 5.—Cross Section Through Solid Floor I-Beam Construction.

In bridges consisting of a number of spans resting on masonry piers, spaced at the most economical distance, the clear space between the ends of main girders should not be less than four inches, and the effective length of the girders is dependent upon this. In steel trestles where the girders rest on towers, the ends of adjoining girders should be placed as close together as the posible expansion will permit, and the effective length will depend on the adopted detail of seat, resting on the cap of the tower posts. In this construction, where the adjoining spans are of different lengths, the different end reactions will produce eccentric loads in the posts, which should not be neglected in the design.

The effective depth of main girders, which is the distance between the centres of gravity of flanges, is usually made to vary from ⅛ of length of span, for short spans, to 1/12 of length for long spans. If, for reasons of clearance, it is necessary to make the depth less than 1/12 of span, additional metal should be inserted so that the deflection will not be more than it would be with the above-mentioned depths. In bridges designed for very light traffic, it is more economical to use shallower girders.

There is a diversity of opinion among railroad engineers regarding the best spacing centre to centre of deck girders. This spacing consequently varies from 6½ feet for short spans to 10 feet for longer ones. Where the track is on a curve this spacing should be increased to accommodate the deviation of the track and to give additional stiffness to resist the centrifugal force of the trains.



The middle ordinate to any chord $= R \text{ versine } \frac{\Delta}{2}$

where R = radius of curve

$\sin\frac{\Delta}{2} = \frac{\frac{1}{2}\text{ chord}}{\text{radius}}$

The centre line of bridge is made to bisect the middle ordinate to the bridge chord.

$a = \frac{1}{2}X - y + \frac{1}{2}W + Z$

$b = \frac{1}{2}X + \frac{1}{2}W + y + k$

where y is the middle ordinate to a chord whose length is equal to the distance centre to centre of kingpins of the car assumed.

$\frac{1}{2}W$ = one half the width of the car

z = the middle ordinate to a chord whose length is equal to the overall length of the car

$k = h \tan\theta$

h being the distance from the base of rail to the top of girder and θ the angle of superelevation

x = the middle ordinate to the bridge chord

Having determined a and b the girders are spaced so that the clearance on either side of the centre line of bridge will be equal to the greater of these two dimensions.

Fig. 6.—Clearance in Through Bridges on Curves.

Through girders are spaced as close together as the government clearance requirements will permit, and when the track is on a curve, this spacing is increased in proportion to the degree of curvature. A simple method of determining this required clearance is indicated in Fig. 6 and its accompanying explanatory note.

Skew in bridges should be avoided wherever possible, but it is sometimes necessary. It is less objectionable in girder spans than in trusses. Even though skew in the face of abutments is necessary, the ends of the girders should be made square to the track, as this is a better arrangement for the deck. The character of the skew is termed right-handed when it is in the direction shown in Fig. 7.

The depth of the deck is the distance from the base of rail to the top of the girder, and when the track is on a tangent this depth is ½-inch less than the depth of tie required to support the wheel loads.

It is a common occurrence for engineers who do not correctly forsee the depths required for bridges, in the deck, in the girders themselves, and in the pedestals, to make mistakes in the construction of the masonry; and to overcome such errors it is often necessary to belly the bottom flanges of girders at the ends. This costs the contractor at least $200 a span for the steel, and is usually a total loss to him.

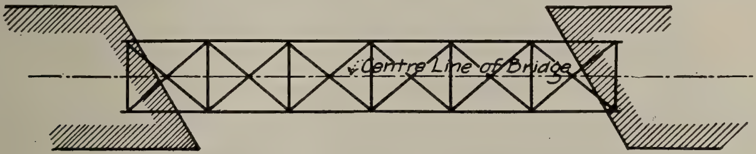
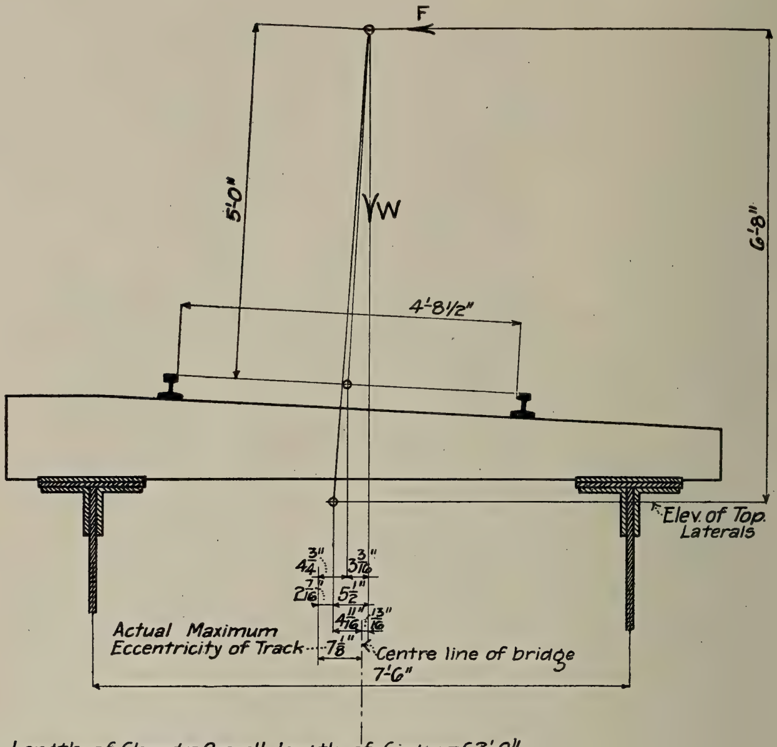


Fig. 7.—Right-Handed Skew, Showing Bridge with Ends Square.

Camber.—This allowance is usually insisted upon for plate girder spans. The intention is to put sufficient camber in the span to more than overcome any possible deflection from loading, because a girder constructed straight might have a reverse camber when erected, which would have an unsightly appearance. A great many bridge engineers consider that this is unnecessary refinement, but if it is specified, good results can only be obtained by proper details and rigid shop inspection. Cases are known where two girders on the same span have reverse camber, although built from the same drawings and in the same shop. Another difficulty in regard to camber is in double track bridges when one track only is loaded. Where there is one centre girder there is more deflection in the outer and lighter girder and where there are only two main girders there is more load carried by the one girder than the other, and therefore unequal deflection exists. The only arrangement to avoid this difficulty is to place two girders under each track, as if they were separate spans. This is a more expensive arrangement, but it is the best design, because the wear and tear on the bridge is



Length of Chord = Overall length of Girder = 62'-0"

Arc = 62·06 ft. Radius 406·424 ft.

$\frac{1}{2}$ Versine = $7\frac{1}{8}$" Elevation of Outer Rail = 3"

C.G. of Arc from vertex $\frac{06}{62\cdot06} \times 406\cdot424 \times 12 = 4\frac{3}{4}$"

Required Eccentricity for Equalized Stresses, train standing $3\frac{3}{16}" + 4\frac{3}{4}" = 7\frac{15}{16}"$

Velocity of Train 30 feet per second

F = ·0688 W Couple due to F = ·0688 W x 80" = 5·504 W

Eccentricity due to F = $-5\frac{1}{2}$"

Required Eccentricity, train moving = $7\frac{15}{16}" - 5\frac{1}{2}" = 2\frac{7}{16}"$

Actual Eccentricity = $7\frac{1}{8}$"

Effective Eccentricity, train moving $4\frac{11}{16}$"

Effective Eccentricity, train standing $\frac{13}{16}$"

Train moving, outer girder has excess $4\frac{11}{16}/90 \times W = 10\cdot42\% \frac{W}{2}$

Train standing, inner girder has excess $\frac{13}{16}/90 \times W = 1\cdot8\% \frac{W}{2}$

Maximum Bending Moments.			
	Live Load	1,950,000	3,169,000
	Impact	1,219,000	
	Eccentricity	10·42 %	330,000
	Dead Load		458,000
	Total		3,957,000

Assumed Loading L.L. Coopers E60
D.L. 1017 lbs. per lin. ft. of Girder

Fig. 8.—Distribution of Loads for Bridges on Curves.

lessened and each half is free to move and deflect by itself when loaded, without straining the other half which would probably be unloaded. It is often an advantage, too, in erection, as one track can be completed first, without interrupting traffic on the other track.

Calculation of Stresses.—In plate girder spans, as in all types of railway bridges, this calculation is based on the following loads: 1, dead load; 2, live load; 3, impact; 4, wind stresses; 5, centrifugal force when track is on a curve, and 6, traction. The methods, both analytical and graphical, for obtaining the dead and live load stresses, are fully covered in all text books on this subject and universally recognized in practice. As these basic methods are unaffected by evolution in construction it seems unnecessary to consider them here.

The dead load per lineal foot of track must be assumed to begin the calculations. It consists of the weight of material in the deck and the steel in the structure. In determining the deck it is usual to consider first the maximum axle load plus impact, distributed over three ties. This will determine the size of ties needed, and as the rails and guards and fastenings are standard, the weight of the deck is then known. The weight of the steel must be assumed either by approximate methods or from records of weights of other spans, and when the design is completed a check should be made to make sure that these assumptions agree with final results.

The live loads adopted by various railroads are usually standard types of consolidated engines followed by a uniform train load and an alternate heavy concentrated load, and the different classes of loading are provided for by varying the loads in the same proportion throughout the whole train. In this way the calculations of stresses are simplified and the results are quite satisfactory, as it is impossible to foresee in every case the exact wheel loads which may be carried by the bridge. Moreover, by the adoption of this standard loading, tables of shears and moments for deck spans of various lengths can be made, which save time and labor in actual practice. In through bridges it is always necessary to calculate each bridge by applying the actual wheel loads.

Impact stresses should always be added to provide for the momentum of the live load caused by deflection of the bridge, the unevenness of the track, vibrations, and various other conditions, but the amount of this impact is always a stumbling block to bridge engineers, as shown by the number of different formulae in use. The intensity of this impact depends on the length of span under load, the proportion of dead and live loads, the elasticity of material and other causes which are unknown, and it is imperative that practical experiments be made on all types of bridges to determine a formula for impact which will be reasonable and satisfactory. However, all formulae used at present are doubtless on the safe side.

The wind stresses in an ordinary plate girder span do not affect the sections of the main girders at all, because they are usually not considered unless they exceed 25% of the other loads combined. In plate girder bridges a moving load of 600 pounds per lineal foot of span is ample to provide for the wind on a moving train, for the oscillation of the train and for the wind on the bridge itself. This force will determine the wind stresses in the lateral system.

The centrifugal force of a train, when the bridge is on a curve is not so important as the preceding stresses, but it cannot be neglected and should be considered, both in the laterals and in the main members. The equatron $F = \frac{W V^2 D}{85700}$ gives the centrifugal force for a load W on a curvature of D degrees with a velocity of V miles per hour. It is considered to act in a horizontal direction, five feet above the base of rail and at right angles to the line of bridge. All engineers agree that the force of F must be resisted by the lateral bracing in the same manner as the wind stresses, but they do not all have the same opinions of the stresses produced in the main girders. The best arrangement of a curved track on a bridge, is to place the axis of the bridge parallel to the chord of the curve, and to make the centre line of the bridge bisect the middle ordinate of the curve at the centre of the span. In other words, the deviation of the track from the centre line of the bridge is the same at the middle as at the ends of the span, and this is an advantage in the event of train derailment. Some bridge engineers are satisfied to consider that, with this position of the track, there will be practically the same load on both girders, and they only add centrifugal force stresses to the lateral system, and use the same girders for a track on a curve as for a track on a tangent.

A more general practice is to assume that there is an overturning action, and that in addition to one-half the live load, there is a load on the outer girder equal to $\frac{F\ h}{b}$, where F is the centrifugal force, h is the height at which F acts above the lateral system, and b is the spacing centre to centre of girders.

Another practice is to consider that the eccentricity of the track at the centre of the span is constant over the entire length of the bridge. Then the proportion of the live load to be added to each girder is given by the formulae $W \left(\frac{M + b}{2\ b}\right)$ where W equals the live load, M is the centre ordinate to the curve, and b is the distance centre to centre of main girders. A thorough treatment of this subject is given in Fig. 8 and the data accompanying it. The theory applied in this example was advanced by Ward Baldwin, who is a college professor and a practical man, and perhaps the best authority on this subject in the United States. He was consulting engineer on a large viaduct in Cincinnati, which the writer designed. In this example the track is super-elevated to suit only a medium speed of train. On account of this arrangement, neither the vertical direct load of a stationary train, nor the direction of the resultant force of a train at high speed, pass through the centre line of track, and these conditions are all taken care of in the calculations. Mr. Baldwin also finds the centre of gravity of the segment, and uses it as the effective position of the centre of the track throughout the length of the span for deck girders. He then arrives at the effective eccentricity for either a fast or slow moving train. The worst condition is used in the calculations, and both girders are made the same section. The extra load which is figured to come on one of the main girders is given as a percentage of the applied load. It is used in the stresses in connection with the live load plus the impact. The above method of determining the centrifugal force stresses, combines all the assumptions made by other engineers, and suggests new ideas which merit consideration. It was the first time, so far as the writer knows, that it had ever been used and it is the more interesting on that account.

The same theory applies in the calculations of the floor system and girders of through spans, with the exception that the position of the centre line of the track at the various panels is used in the calculations, and it is not necessary to assume an effective position of the track over the length of the bridge. In through bridges, too, the stringers should be made to follow the line of track as near as practicable.

When the track is on a spiral, the degree of curvature at the centre of the span is considered constant over the whole length of the bridge in figuring the centrifugal force.

The traction stresses are caused by the momentum of trains either starting or stopping, the amount being dependent upon the coefficient of friction between the wheels and the rails, which is usually considered .20. This force is not considered in connection with the main girder at all, but it does affect the lateral bracing in through plate girder spans. The best construction, to provide for this force, is to connect the bottom of the stringers to the lateral angle at the points where they cross, and to add a diaphragm between the stringers at these points, which will take the resultant of the stresses in laterals and stringers. This diaphragm will also act as a lateral support for the top flange of the stringers. The reason for the above detail is to prevent the traction from producing any horizontal bending moment in the floor beams.

PART II.

Consideration of Sections of Material — The Framing of the Various Sections — Shipment and Erection of Plate Girder Spans—Adaptability of Design to Shop Practice and Erection Conditions

The first part of this article, after enumerating the various advantages of plate girder construction, for example, simplicity and uniformity of construction, solidity and durability, speed in erection, and low office cost, went into the calculation of the stresses due to the loads. The methods by which the ordinary dead and live loads are obtained do not form a part of the article as the reader may refer to any authoritative text on the subject for them. Following, the various sections of material to be used in design are next in consideration.

The web plate is first determined, as its depth is already decided, and 100,000 pounds per square inch of gross section is the unit which is generally used in obtaining the required thickness of web. It may, however, be necessary in some cases, to use a thicker web than is required by this unit in order that the end shear may be transmitted to the flange angles in a length equal to the depth of the girder. This is only probable when the web is connected to the flanges by single lines of rivets and the number of rivets is, therefore, more limited.

The flanges of girders are determined, as in trusses, by dividing the total bending moment at any point by the product of the effective depth and the unit tensile stress. The difference between girders and trusses is that the flange stress varies gradually throughout the length of the girders, while in trusses it is constant between panel points. The bottom flange is usually figured at 16,000 pounds per square inch of net section, and the top flange is made the same total gross section as the bottom, with the restriction that the unsupported length of the flange must not exceed twelve times its width. This limits the spacing of floor beams and cross-brace frames.

As the centre of gravity of flanges cannot be known until the sections are determined, a great amount of time is usually wasted in assuming a centre of gravity and making approximate calculations, and this operation repeated several times until the section is finally settled. Table 1 gives a useful table for railway girders. By use of it the required flange can be determined by one calculation and the labor of figuring the net sections and centre of gravity is avoided.

The effective depth used should never be greater than the distance, back to back, of flange angles.

There is a wide diversity of opinion among engineers as to whether part of the web should be counted as flange area. For this to be so, the flanges and web must act as a solid beam and theoretically 1/6 of the gross section of web would act as flange section, but it is considered to be ⅛ on account of deduction of rivet holes.

Since the bending stresses in the web do not act at any one point, but over the total depth of the web, having a maximum intensity at the edge of the web and zero at the centre, it is very difficult to properly splice the webs to take care of these stresses. It involves a great many rivets and extra material, which, in cases of heavy girders, is worth more than the material saved in the flanges. Moreover, if the web is considered to take tension it is good practice to plane its edges to avoid tearing, and this is additional expense.

In some cases where the web does not need to be spliced, owing to possible lengths that can be procured from the mills, there would be a saving of material and cost, to consider ⅛ of the web as flange section. But, since it is only an assumption that the girder acts as a solid beam, and since the flanges are figured the same as trusses, it seems advisable to consider that the web resist shear only; the only difference being that the web acts continuously instead of at panel points. The top flange of deck plate girders is often made an H-shaped section of four angles and side plates. The main advantage of this section is its suitability to easy framing of the deck ties, as there are no rivets or cover plates to be taken care of. The objections to such a section are, that it is more difficult to manufacture, because extra drawings are necessary and the shop work is more expensive; this for the reason that in modern shops equipped with spacing tables, it is a big advantage to have both flanges similar. Moreover, the H-flange is not as strong to resist buckling from handling and from compression stresses, as is the more common section composed of two angles and cover plates. Hence, if it is used, the brace frames should be made with the top strut the same depth as the flange section. This is necessary because the

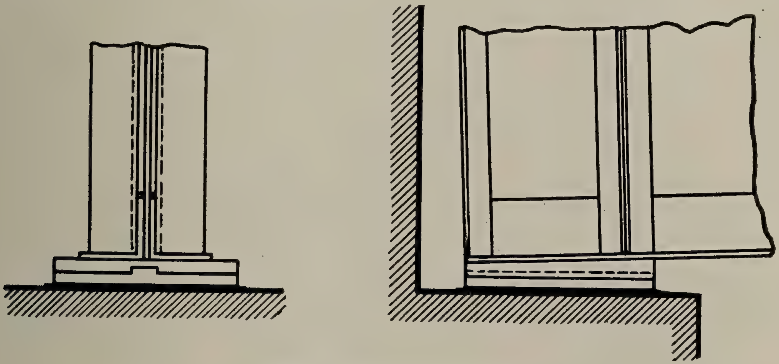


Fig. 9.—Sliding Bearing.

top laterals are at the elevation of the bottom of the H-section, which is some distance below the deck, and because the web stiffener angles must break at the bottom of the H-shaped flange, which is a marked weakness.

Stiffener Angles.—These are an important factor in the strength of girders, and, as their duty is not generally understood, they deserve some explanation. They may be divided into two classes, viz., those that resist concentrated loads and those which merely prevent web buckling. The end stiffeners transmit the total end reaction to the web plate where it is converted into shearing stresses. As these stiffener angles are ground to fit the curved fillet of the flange angles, it is evident that a large part of the reaction is transmitted to the stiffeners by direct bearing of the outstanding leg on the flange angles, and it is therefore necessary to have this leg as wide as the flange angles will permit, and care should be exercised in the shop to ensure that these angles have an exact fit and true bearing. In arriving at the sections of end stiffeners, it is usual to consider that they act in conjunction with the filler plates, as a column of length equal to the depth of the girder, and that the end reaction produces a compression which is constant throughout the length of the column, whereas in reality, the stress diminishes from a maximum at the bottom to zero at the top. As the filler plates cannot be considered to bear on the flange angles in actual practice, the stiffener angles must take the entire stress at the break

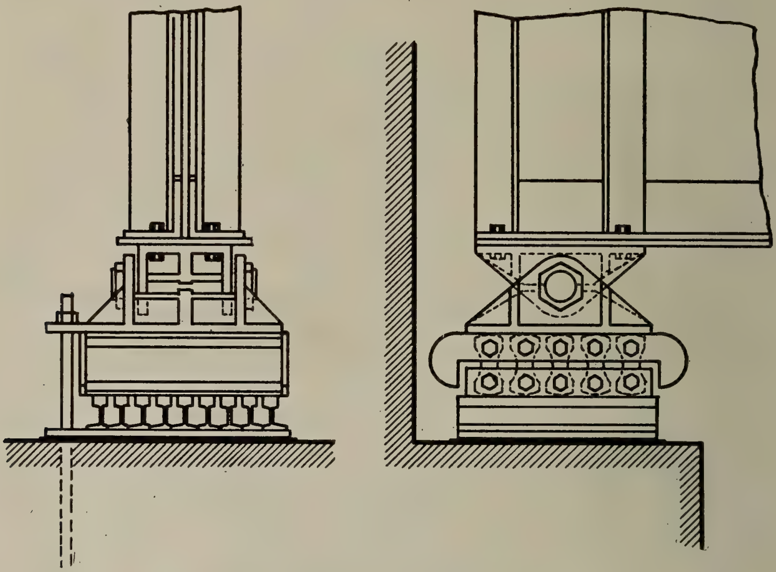


Fig. 10.—Pin Bearing for Expansion End.

between the filler and toe of the flange angles; and it is at, or below this point, that the failure of end stiffeners would usually occur. Taking this into account, and also the fact that the bearing of the outstanding leg does not transmit all the end reaction, but that part of it will go into the other leg through bearing on the fillet and rivets at the bottom, it is a common practice to use end stiffener angles, the gross section of which is considered to resist the reaction at 12,000 pounds per square inch. In addition to these considerations, the designer must use judgment in the selection of the angles, so that the outstanding leg will be thick enough to prevent its buckling near the bottom, where it is highly stressed. There should also be sufficient rivets connecting these stiffeners to the web to transmit the reaction intended.

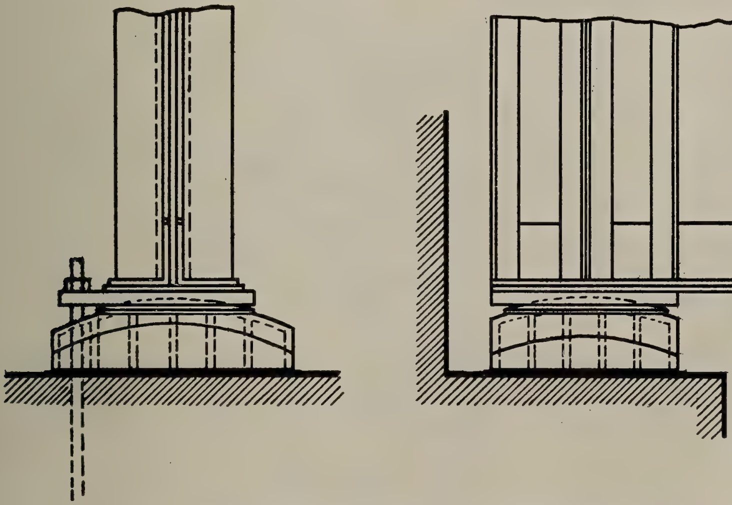


Fig. 11.—Disc Bearing for Fixed End.

Intermediate stiffeners which are merely intended to prevent the web plate from buckling, do not cause the designers of girders much worry, as all railroads have adopted standard sections for different depths of girders and certain limitations as to the spacing of them, but it is a satisfaction to know that their failure in acual structures has been very rare. They have been generally determined by formulae based on compression in the web plate due to shearing forces acting upon it. This seems the most reasonable method. The spacing of these intermediate stiffeners should therefore depend on the shearing stresses, and this would necessitate their being spaced closer toward the end of the girder.

Theoretically there is no reason for the intermediate stiffener angles to bear on the bottom flange, but it is always done in practice and makes a neater job. It is necessary, however, in the top flanges because it tends to prevent the outstanding leg of the flange angles from buckling and in deck girders it helps to transmit the loads from the track into the web plate.

Pier Members.—They are determined in area, by the unit, 400 pounds per square inch for concrete masonry. For spans up to 75 feet in length sliding masonry plates only are used, as indicated in Fig. 9, with a tongue and groove to resist side motion. The groove should be on the upper shoe plate as shown, so that dirt and water cannot lodge in it.

With this type of bearing it is advisable to consider that the stiffeners over the inside edge might have to take 75 per cent. of the total end reaction, because any deflection in the span tends to move the end reaction to this point.

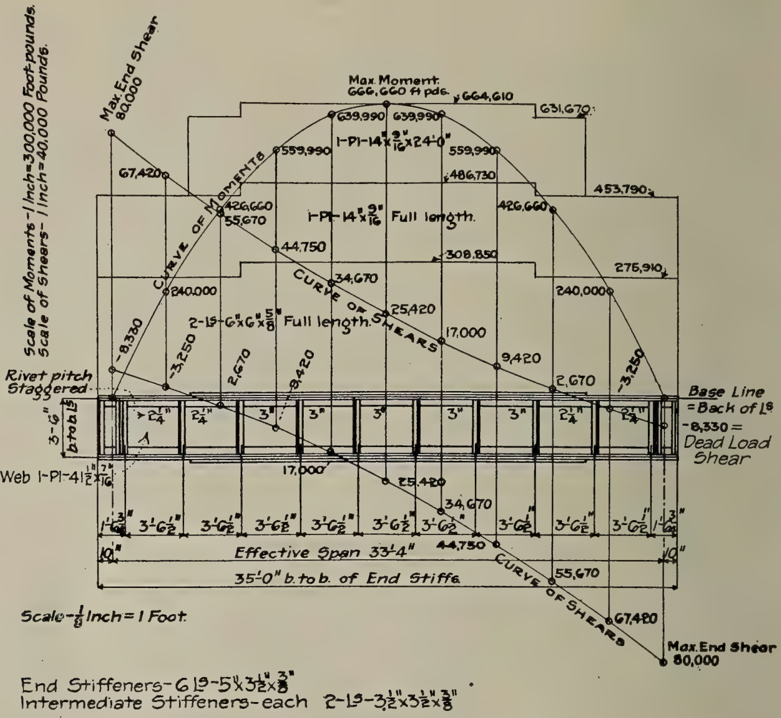
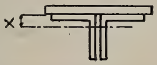


Fig. 12.—Stress and Section Diagram for Deck Plate Girder.

For spans 75 feet long or over, pedestals are necessary to properly distribute the bearing pressure, and to adjust themselves to deflection and camber. Fig. 10 indicates the pin type which is generally used in the United States, and it may be made up of structural shapes, or cast steel. By means of the pin the bridge is securely anchored to the bottom pedestal.

DISTANCE OF CENTRES OF GRAVITY OF PLATE GIRDER FLANGES FROM BACK OF ANGLES

Flange Area does not include 1/8 of web
Net Area – 2-6"x6" L^s 4-7/8"$^\phi$ rivets off
2-8"x8" L^s 6-7/8"$^\phi$ rivets off
Cover Pls. 2-7/8"$^\phi$ rivets off



2-L^s-6"x6"x5/8"				2-L^s-6"x6"x11/16"			
Cover Pls.	x (ins.)	Gross Area sq. ins	Net Area sq. ins.	Cover Pls.	x (ins.)	Gross Area sq. ins.	Net Area sq. ins.
None	1·73	14·22	11·72	None	1·75	15·56	12·81
14"x3/8"	1·21	19·47	16·22	14"x3/8"	1·26	20·81	17·31
14"x7/16"	1·14	20·35	16·97	14"x7/16"	1·19	21·69	18·07
14"x1/2"	1·07	21·22	17·72	14"x1/2"	1·13	22·56	18·81
14"x9/16"	1·01	22·10	18·47	14"x9/16"	1·07	23·43	19·56
14"x5/8"	·95	22·97	19·22	14"x5/8"	1·01	24·31	20·31
14"x11/16"	·89	23·85	19·97	14"x11/16"	·95	25·18	21·06
14"x3/4"	·83	24·72	20·72	14"x3/4"	·89	26·06	21·81
14"x7/8"	·73	26·47	22·22	14"x7/8"	·79	27·81	23·31
14"x1"	·63	28·22	23·72	14"x1"	·68	29·56	24·81
14"x1 1/8"	·53	29·97	25·22	14"x1 1/8"	·59	31·31	26·31
14"x1 1/4"	·43	31·72	26·72	14"x1 1/4"	·49	33·06	27·81
14"x1 3/8"	·34	33·47	28·22	14"x1 3/8"	·40	34·81	29·31
14"x1 1/2"	·25	35·22	29·72	14"x1 1/2"	·31	36·56	30·81
14"x1 5/8"	·16	36·97	31·22	14"x1 5/8"	·23	38·31	32·31
14"x1 3/4"	·08	38·72	32·72	14"x1 3/4"	·14	40·06	33·81
14"x1 7/8"	b. of L^s	40·47	34·20	14"x1 7/8"	·06	41·81	35·31
14"x2"	out	42·22	35·72	14"x2"	out	43·56	36·81

2-L^s-8"x8"x11/16"				2-L^s-8"x8"x3/4"			
None	2·25	21·06	16·93	None	2·28	22·88	18·38
18"x3/8"	1·66	27·81	22·93	20"x1 3/4"	·37	57·88	49·88
18"x1/2"	1·50	30·06	24·93	20"x1 7/8"	·28	60·38	52·13
18"x5/8"	1·36	32·31	26·93	20"x2"	·19	62·88	54·38
18"x3/4"	1·23	34·56	28·93	20"x2 1/8"	·11	65·38	56·63
18"x7/8"	1·10	36·81	30·93	20"x2 1/4"	·03	67·88	58·88
18"x1"	·98	39·06	32·93	20"x2 3/8"	out	70·38	61·13
18"x1 1/8"	·87	41·31	34·93	20"x2 1/2"	out	72·88	63·38
18"x1 1/4"	·76	43·56	36·93	20"x2 3/4"	out	77·88	67·88
18"x1 3/8"	·66	45·81	38·93	20"x3"	out	82·88	72·38
18"x1 1/2"	·56	48·06	40·93	18"x1 1/2"	·64	49·88	42·38
18"x1 5/8"	·47	50·31	42·93	18"x1 5/8"	·55	52·13	44·38
18"x1 3/4"	·38	52·56	44·93	18"x1 3/4"	·46	54·38	46·38
18"x1 7/8"	·29	54·81	46·93	18"x1 7/8"	·37	56·63	48·38
18"x2"	·20	57·06	48·93	18"x2"	·28	58·88	50·38
18"x2 1/8"	·11	59·31	50·93	18"x2 1/8"	·19	61·13	52·38
18"x2 1/4"	·03	61·56	52·93	18"x2 1/4"	·10	63·38	54·38
18"x2 3/8"	out	63·81	54·93	18"x2 3/8"	·02	65·63	56·38
18"x2 1/2"	out	66·06	56·93	18"x2 1/2"	out	67·88	58·38
18"x2 3/4"	out	70·56	60·93	18"x2 3/4"	out	72·38	62·38
18"x3"	out	75·06	64·93	18"x3"	out	76·88	66·38

Nearly all Canadian railroads use the disc bearing, as shown in Fig. 11, and it is superior to the pin type in many ways. It is sure to have a true bearing, as it adjusts itself in all directions to suit uneven masonry and deflection in the span. It is also convenient in erection as the girders can be dropped into place with a minimum trouble in adjustment.

In the disc type most of the end reaction should be considered to enter the girder over the centre of the disc with a small proportion at each edge, while in the pin type, where the shoe is deeper, all the end stiffeners can be considered to take an equal share.

Expansion in Girders.—This condition should be provided for to the extent of one inch for each 80 feet of span, and for all spans over 75 feet in length roller nests must be used at one end. These should be constructed so that the sides may be taken off and the rollers cleaned and oiled; also in such a way that they will shut out as much dirt as possible and not hold water. Fig. 10 represents a good design for roller nests, in which segmental rollers are indicated, the lower bearing surface being a nest of rails. The segmental rollers are convenient because they occupy a minimum space for a large diameter. The rails are important because dust will not lie under the rollers, but will fall through, affording easy removal. Bearing plates should always be made thicker, to allow for rust of ½ inch in thickness.

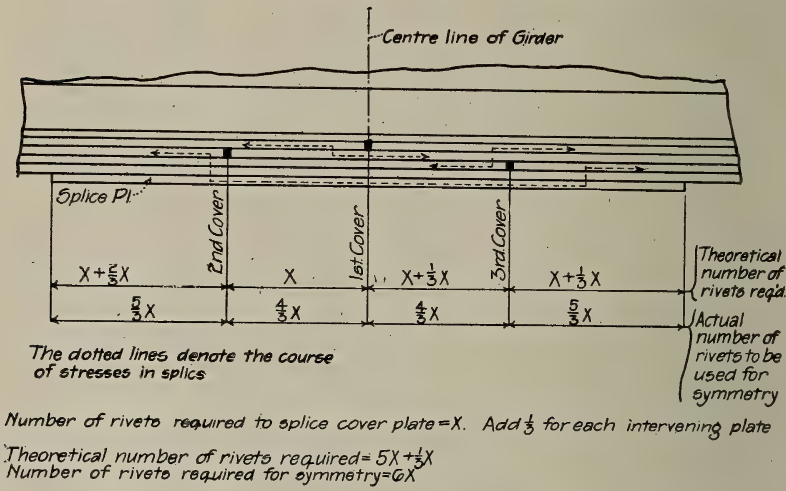

Centre line of Girder
Splice Pl.
2nd Cover
1st Cover
3rd Cover
X+2/3X
X
X+1/3X
X+1/3X
5/3X
4/3X
4/3X
5/3X
Theoretical number of rivets req'd.
Actual number of rivets to be used for symmetry
The dotted lines denote the course of stresses in splics
Number of rivets required to splice cover plate=X. Add 1/3 for each intervening plate
Theoretical number of rivets required= 5X+1/3X
Number of rivets required for symmetry=6X

Fig. 13.—Flange Splice.

Lateral Angles.—Usually these are made heavier than the stresses require, to ensure stiffness of the bridge. In deck spans the entire lateral forces are considered to be carried by the top laterals, through the end brace frames to the anchors, while the bottom laterals are merely considered to stiffen the bottom flanges against wind. The latter are often omitted in deck spans under 50 feet long, as the interior brace frames give sufficient stiffness to the bottom flange.

In through plate girder spans there is only one system of bracing, which necessarily carries all the lateral stresses, and the traction stresses in addition.

When rods are used as laterals, an initial stress of 10,000 pounds should be added to the other stresses to take care of the tension induced in the rods by tightening the nuts or turnbuckles.

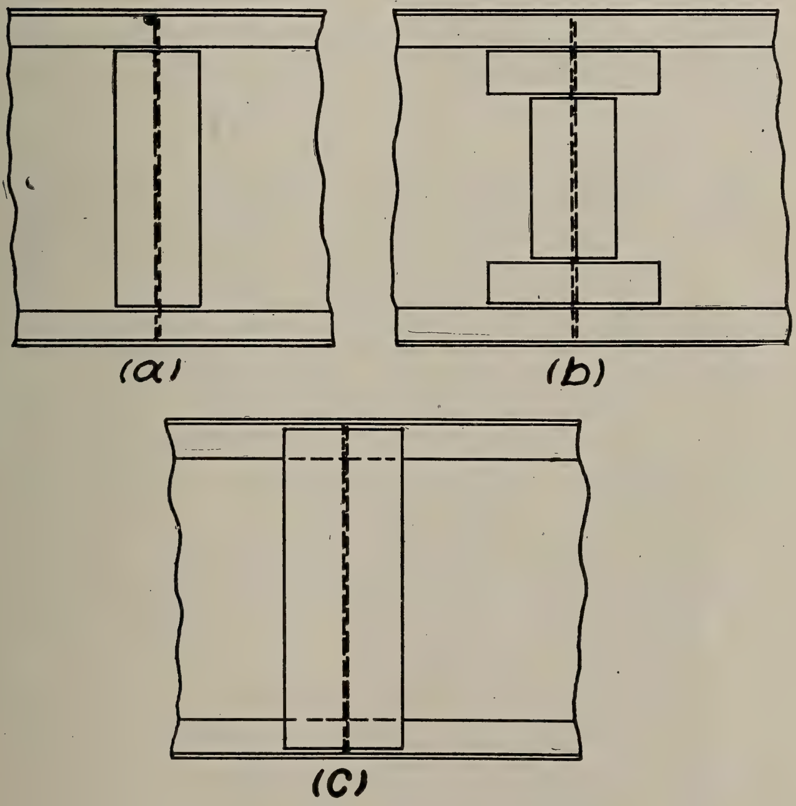


Fig. 14.—Plate Girder Web Splices.

Stringers and Floor Beams.—These constitute the floor system in through plate girder spans. As the stringers are merely deck girders of reduced length, their design is similar. They should be made as deep as practicable, and should be riveted to the webs of the floor beams. Two lines of stringers per track is good practice, but four lines are more able to resist derailed trains, and do not require such large ties. With four lines of stringers, the common method is to assume that the inner and outer stringers take their share of the wheel loads inversely in proportion to the distance they are spaced from the rail. As a matter of fact, when the rail is midway, the inner stringer gets a greater proportion of the wheel load, because the ties are continuous beams over four supports. Some account should be taken of this.

Floor beams should also be made as deep as practicable. When the top flange is of moderate width it should be about one inch below base of rail, so that it will support derailed trains; but when the top flange is 18 in. or 20 in. wide, it is good practice to drop it three inches below the base of rail, so that a wood strip can be placed on it to support the rail, as well as to be useful in the event of wheels passing over it.

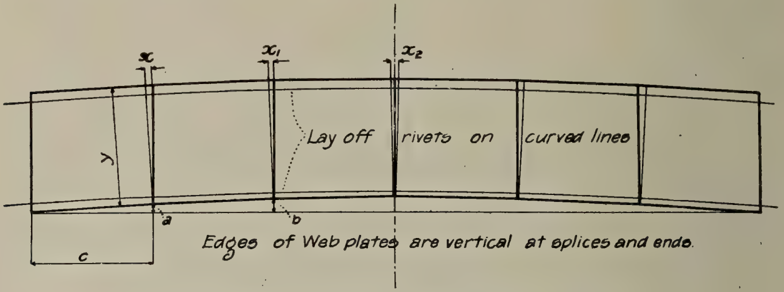

x
x₁
x₂
Lay off rivets on curved lines
y
a
b
c
Edges of Web plates are vertical at splices and ends.

Fig. 15.—Plate Girder with Five Web Splices.

Stress and Section Diagrams.—Such diagrams are convenient for reference on bridge plans, indicating graphically the bending moments and shears along the length of the span, and the flange and web sections which correspond. Fig. 12 shows a complete form, and needs little explanation. The two shear diagrams represent the change in web shear as the train passes over the bridge in either direction, the live load and impact shear changing, while the dead load shears remain unchanged. These curves determine the required web section and the web flange rivet spacing.

The bending moment diagram determines the required flange areas and the length of cover plates. These plates are made to pass beyond the curve far enough to develop the strength of the plate in rivet connections. When the flange rivet spacing is less than 2½ inches, staggered, it is considered that two extra rivet holes are deducted from the net flange area, which explains the drop in the area lines shown in the diagram. Where web and flange splices are necessary this diagram is convenient for reference in getting out detail drawings.

The Design of Trough Floors.—The computation is different from that of regular plate girders, as the section is determined by their net moment of inertia, as in the case of a solid beam. When the troughs run parallel to the line of track, the axle loads are assumed to be distributed in a width of nine feet, which is the length of the ties, and this decides the amount of load carried on each trough. When the troughs run normal to the line of track, the axle load should be considered distributed over three ties of nine feet in length. A minimum thickness of 7/16 inch is recommended in trough floors to resist rust, and the surface to be covered with concrete should be left unpainted in order that the concrete will adhere better.

Fabrication of Sections.—The details in connection with the framing of the various sections of a bridge may be the cause of its failure, as the strength of a structure is measured by the strength of its weakest point. It is necessary for the detailer to have an intelligent grasp of the action of stress through rivets and sections and he must keep in mind the sizes and maximum lengths of material that can be procured from the mills. He must be familiar with shop practices and erection methods, and must also consider the conditions of exposure of the bridge in actual service.

Splices should be avoided where possible, by the use of the longest angles and plates that can be obtained. Flange splices should alternate except in special field splices where it is necessary to have a compact joint. The theory connected with flange splices can be best explained by an example and the reasoning used in it can be applied in all cases. In the example shown in Fig. 13 the flange consists of two angles and four cover plates, and it is required to splice the two angles and three plates. The first cover plate breaks at the centre of the girder, and the second and third at equal distances each side of the centre. For convenience, consider the cover plates to be of equal thickness, and a splice plate added on the outside, thick enough to splice any one plate. Let X denote the number of rivets, in single shear, required to develop each plate, and let the arrow lines represent the lines of stress from one plate to another. When the stresses must pass through intervening plates, it is good practice to increase the number of rivets required by 33 1/3 per cent. for each intervening plate, because the rivets are less efficient on that account. The number of rivets required theoretically is then obtained, as explained in the example, and in order to make the girders symmetrical about the centre line, extra rivets are added. To complete the splice the best arrangement is to break the flange angles just beyond the cover plate splices. They should alternate, one on each side of the centre line of girder. The number of rivets are obtained in the same way as for the cover plates. Where space is limited, the angles could be spliced in conjunction with the cover plates, but the method outlined above is better, because it does not require the use of such long rivets

The Splicing of Web Plates.—This is a difficult detail and a subject of disagreement among engineers. If the web is considered to transfer shear only, the best solution for the splice is to use a pair of splice plates of a depth equal to the clear distance between the flange angles, and attached with the necessary number of rivets, to properly develop the splice. This arrangement is indicated in Fig. 14 (a). If it is desirable to have the web carry its due proportion of the bending moment, this form of splice can also be used, but it is necessary to have a sufficient number of rivets in the connection to resist the web shears and also the amount of bending moment attributable to the web plates. This requires a large number of rivets and their value varies directly with the distance from the centre of the web plates.

A common form of splice is shown in Fig. 14 (b), in which the horizontal plates adjacent to the flange angles are considered to resist the bending moment, and the vertical plates to resist the shear. This is perhaps the cheapest arrangement, but not the best, for it concentrates the bending stresses at two points, whereas, in reality they occur over the entire depth of the girder; and it splices the web for shear, in a depth which is considerably less than the web, and thereby introduces unknown internal stresses. The horizontal plates are sometimes made to overlap the flange angles, but this arrangement has the same objections.

The splice shown in Fig. 14 (c) is recommended by some engineers as the best form of detail. It is similar to (a) except that the splice plates are extended over the flange angles and either crimped or placed on fillers. In this way the splice has the same depth as the web itself. This is expensive shop work and requires a lot of rivets. It has the additional objection that the flange rivets in the splice are used in a double capacity, as they act with the splice and also have the same work to perform as all flange rivets.

Since there is evidently considerable uncertainty in the efficiency of web splices, where the web is considered to take bending stresses, it seems safer, as stated before, to conider that the web takes shearing stresses only. The best method, and one which is used by the Pennsylvania Railroad, is to use a splice which is considered sufficiently strong to develop the bending stresses, if ⅛ of the web acts as flange section, but to disregard the web in figuring the flange, by using sufficient flange section to resist the entire flange stress. In this way the factor of safety of the bridge is increased at very small cost, and this provides for future excessive loading.

Rivet Spacing.—The rivet spacing in flange angles is dependent upon the assumption that the shear at any cross section is transmitted into the flanges in a length equal to the depth of the girder. This depth should be considered the distance, centre to centre, of web flange rivets.

In the top flange of deck girders, there is also the direct load from the track to be considered, which makes the calculation more complicated. Some engineers consider that the axle load plus impact is supported by only two ties, but it is more common practice to consider that it is distributed over three ties. For an example, consider the case where none of the bending stresses is carried by the web plate, and assume that the direct load is distributed over 36 inches. As this distance is very short, the impact should be counted 100 per cent. of the live load. If W represents the combined wheel load plus impact, then W/36 will equal the vertical load per lineal inch on the flange rivets; and if S represents the shear in the girder at any cross section, S/h will equal the horizontal shear per lineal inch, where h is the depth.

The resultant stress on the rivets per lineal inch of flange will be $\sqrt{\left(\frac{W}{36}\right)^2 \times \left(\frac{S}{h}\right)^2} = K.$ The working value of one rivet in bearing on the web, divided by k, will give the required pitch of rivets in the web flange. This spacing is slightly less where ⅛ of the web acts as flange section because the entire shear does not have to be transmitted through the flange rivets.

The rivets in cover plates are spaced so that there will be a sufficient number between the ends of two successive plates to develop the strength of the longer one. This, however, usually gives too great a pitch, as practical reasons require a maximum allowable spacing in order that the surfaces will be held in even contact.

Camber in Girders.—As stated before, this is often specified, but it is only obtained by proper details. Where there are no web splices in the girder, it is necessary to punch the flange holes in the web plate in the form of a circle, and the flange angles are pulled down to suit, when the girders are assembled in the shop.

When the web plate has splices at various points there are two methods of camber commonly used. In the first method, which is the cheaper and more common, the top and bottom flanges are considered as circles about a common centre. This is accomplished by adding the extra length required in the upper flange, at the points of web splice. This extra length is the same at each splice, and the web splice plates are detailed to suit.

The other method, which is used by the Pennsylvania and other railroads, is more to be commended, and is shown in Fig. 15. Here the upper and lower flanges have an equal radius and are of equal length, the ends of the girders being vertical. This arrangement has a big advantage in details and shop work. In the example given, the girder is 88 feet long over all, and requires a camber of 1 1/16 inches when figured at 1/1000 of the span. This would be increased by ⅛ of an inch for each web splice to allow for shop play, which makes a total required camber of 1 11/16 inches. For even figures 1¾ inches is considered.

The ordinates, a and b, are then figured at the various splice plates. By proportioning similar triangles, $X = \frac{a}{c} \times y$, and similarly X1 and X_2 are obtained. They diminish in value towards the centre of the girder. The web plates should be planed to these figured bevels, so that they will bear along the splice, and the splice plates are also detailed to suit. If the last method is used, and properly followed in the shop, good results should be obtained.

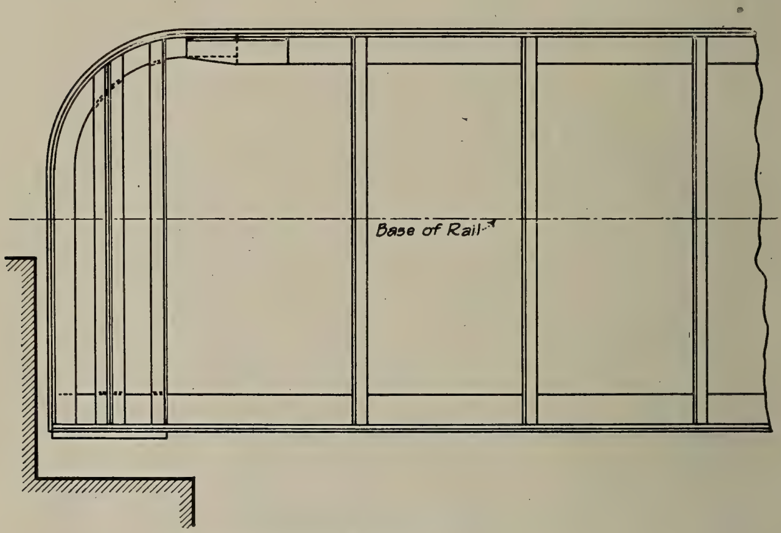


Fig. 16.—Curved End for Through Girder.

Many engineers put the same camber in the deck as in the girders, by using the same dap in the ties along the length of the span. The better practice is to take all camber out of the track for spans up to 75 feet, and to take out only one-half of the camber for greater spans. This for the reason that the bridge looks better to have the track level under dead load only, and moreover, as there is always an excessive camber over that required to balance actual deflection from the ordinary fast trains, these trains would only produce a very slight dip in the track in passing; whereas, if the track had excessive camber, there would always be a bump which would be very objectionable in trains at high speed. With the heavier trains passing there might be a noticeable deflection, which would cause a dip in the track, but the heavy trains would be moving comparatively slowly, and this would not be a bad feature.

The ends of girders in deck spans are necessarily square at the top, in order to support the ties, but in through spans it is customary to curve the top flange for appearance. One method, as indicated in Fig. 16, is to round the corners in the form of a circle with a radius equal to 1/3 the depth of the girder. This is commonly done, and is a good detail as the web plate is not weakened, but the shop work is expensive as the angles must be bent hot and spliced.

Fig. 17 represents a detail which is a cheaper curve to make, as the radius is about 1½ times the depth of the girder. It is also more suitable to resist derailed trains, and cases are actually known where trains have hit the ends of such girders and skidded back on the ties. The main objection to this type of end is, that the web is shallow where the shearing stresses are greatest and it is necessary to reinforce the web by side plates as indicated by "A" in the sketch. In this construction, and also in the case of girders where the bottom chord is bellied, an error is often made by detailers, as they neglect to add the horizontal plates marked "B" in the diagram. In other words, they reinforce the web against rupture from shear in a vertical plane, and forget that, in order to transmit the shear into the flanges in a length equal to the depth of the web, it is necessary to reinforce the web also against rupture in horizontal lines at the toes of the flange angles.

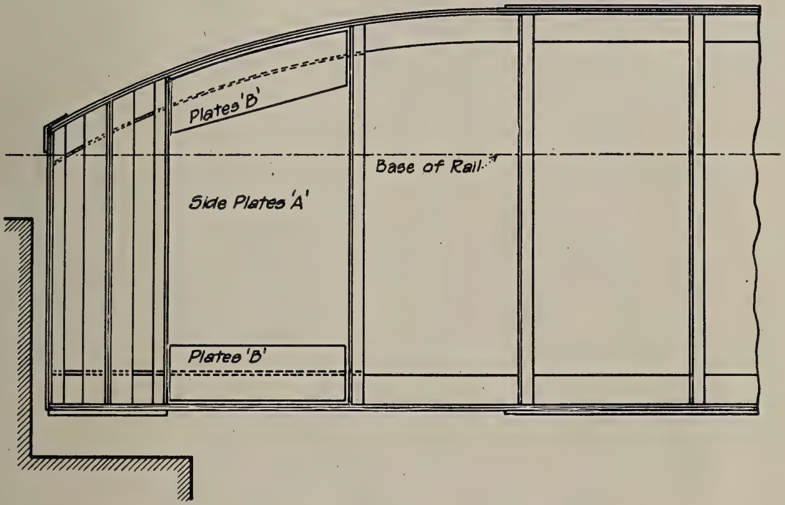


Fig. 17.—Curved End for Through Girder.

The grade of a track does not affect details of plate girder spans, except the sliding bearing plates of smaller spans, in which case either the upper shoe plate or the loose masonry plate must be planed on the bevel. Where the loose plate is bevelled the sliding plane is in the same direction as the expansion, which is an advantage where the grade is steep and the span is long; because if the upper shoe were bevelled there might be considerable rise and fall in the elevation of the track, when the bridge expands.

For smaller spans and light grades the best arrangement is to bevel the upper shoe, and thus keep the sliding surfaces horizontal, as shown in Fig. 9. Thus any tendency for the span to creep on a sloping surface is avoided.

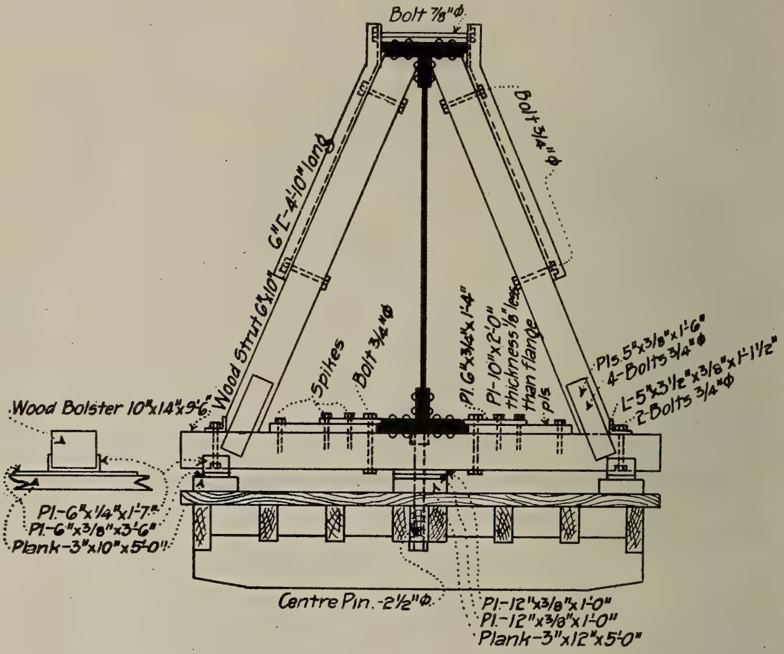


Fig. 18.—Method of Loading Girder on Flat Car.

Modern Methods in Shop Practice.—Shop methods, as well as the action of stresses and the strength of connections, should be well understood by the designer and detailer. Rivets should be spaced to suit the requirements of the spacing tables and multiple punches in common use, and which are specially adapted to plate girder construction. In this way the necessity for templates is reduced to a minimum and the shop costs are lowered.

Bridges should be so made that there is easy access for painting of all parts and no openings or pockets should be left where dirt and water might lodge.

In shipment, plate girders are sometimes troublesome and more expensive than other forms of lading. Where the girders are short and shallow and can be loaded on one car, the method of loading is not very different from any form of heavy material. Where the girders are over sixty feet long and extend over two or more cars, it is necessary to adopt special pivoted bolsters in order that the cars will pass freely around curves. The bolster adopted by the Master Car Builders' Asociation, is recommended in practice and is explained in Figs. 18 and 19. The sizes of material given are for girders weighing between 30,000 and 70,000 pounds, and vary slightly for other weights.

The weight of loading that can be placed on each car, where the load rests on one bearing point per car, varies from ½ to 2/3 of the capacity of car, depending on whether it is located at the centre or whether it is located midway between the centre and the position of the truck, and also depending on the construction of the car. The height of an ordinary flat car above the top of rail, is four feet and two inches, and this, in conjunction with the bolster, should be deducted from the minimum allowable overhead clearance along the line of shipment, in order to find the maximum depth of girder that can be shipped to various sites.

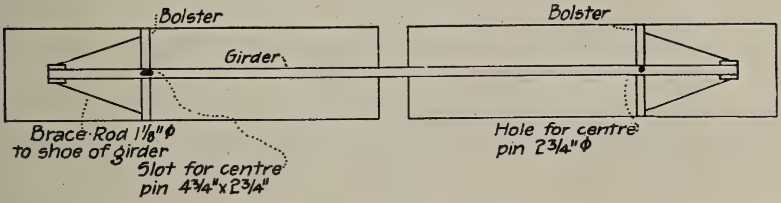


Fig. 19.—Plan of Loading.

The Erection of Plate Girder Spans.—This is accomplished with less expense and in shorter time than in the case of any other form of bridge. As it is an inestimable advantage in railroad construction to replace existing spans with no interference with traffic, the simplicity in erection of plate girder spans by modern methods is most important.

The old methods of erecting girders, where there was no falsework, was to place a gin pole in the centre of the span and gradually work the girders from the bank into position. Where there was falsework in place, which had been necessary for further construction of the road, there were several familiar methods employed. One was skidding the girders on the falsework and lowering by jacks. Another method was placing gantries at the abutments, or on cars, and from these gantries a set of falls was suspended which raised the girders off the cars, after they had been run out on the falsework, and which then lowered the girders into position. Lever cars and wood travellers were also used to lift the girders upon the falsework, after which they were pulled over into position.

The above-mentioned methods were dangerous and slow, and the modern erection car completely equipped, and properly manned, will take a plate girder span which is all riveted up, and drop it into position in less than one-half hour. Some companies use travellers which run on special wide-gauge tracks, but they require special provision in the design of the bridge to provide for this. The Hamilton Bridge Works Company, Limited, have eight erection cars in operation, which run on regular gauge tracks and, therefore, require little delay in rigging up at the site, requiring no provision in the design of the bridge, except that it is necessary to consider the load on the front trucks of the car. This very rarely affects the design, however.

Another advantage in the erection of plate girder bridges is that there are very few field rivets to be driven owing to the fact that the girders can be shipped completely riveted up. This is a big saving in costs, and is stronger construction, since rivets can be driven better in the shops, where machines are power-driven.

The evolution in plate girder bridges has been very pronounced since the year 1850, when they were first built of iron, and the principles of economy in design have changed to a great extent. It is impossible to foresee what changes will develop in the future, in regard to the limiting sizes that will be practicable; the composition and strength of metal that may be used, and the facilities for fabrication and erection that may be invented. For these reasons, the engineer in practice should be daily reminded that no set of specifications can cover all the questions he will have to decide, and that "The most perfect system of rules to insure success must be interpreted upon the broad grounds of professional intelligence and common sense."

APPLIED SCIENCE

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EDITORIAL

UNDERGRADUATE AND GRADUATE

Probably before the next issue of "Applied Science" reaches our readers the corridors of the "Old School," now wearing a dismal and doleful expression in waiting for the return of their natural "toike-oike" element, will be welcoming back hundreds of familiar faces, each one beaming at the possible opportunities to wear off some of the superfluous energy accumulated during the vacation months. Certainly, nowhere is the glad hand of good fellowship more earnestly extended (especially to the freshmen) than among the school men, who, scattered to all parts of the land during the summer, return to renew old associations and resume old activities.

Is there a graduate, who, when he reflects upon the good old days he spent in the "Little Red School," does not feel the tingling touch of youthful vigour and enthusiasm stirred within his veins? The years which one spends at college serve as a seasoning to his future years. After all we are inclined to believe that the associations which one cultivates while an undergraduate should be one of his greatest assets during his professional career. It is the remembrance of such associations that holds the graduate in touch with the school and the Engineering Society.

It is advantageous to the undergraduate to cultivate associations with the graduates in order that he may acquire a broader and more reliable conception of the various spheres and stern realities of the profession, than he otherwise would acquire. Too often undergraduates in engineering do no more to prepare themselves for the engineering profession (apart from their academic work) than for any other profession, and as a result they find themselves sadly handicapped on graduating.

It is equally desirable that the graduate should keep in touch with the undergraduate and the "School," for if he displays interest and confidence in the school and her progeny he will inspire confidence in the world at large. In unity there is strength, and if every graduate is to reap the benefits in store for him, then every graduate must be a booster.

ENGINEERING AND THE NATIONAL EXHIBITION

Once again the great National Exhibition marks the advancement made in our Dominion by the wheels of progress. From far and near vast crowds gather to learn of the improvements accomplished through the ingenuity and applied diligence of man, and disperse again inspired with a new confidence and a new ambition to do greater things. Were it not for this inspiring influence on the public the National Exhibition could not be rated at its present value, but as a result of the increased national pride and ambition stimulated in her citizens, Canada is immeasurably benefited.

The many advances in methods and results are the outcome of the combined efforts of all classes of citizens to be of some service to mankind. The engineer naturally meditates concerning what part his profession has played in making possible such a display of national achievement. Even the engineer himself does not always realize to what extent his work is interwoven with the improvements and inventions on exhibition. Can we then wonder that the public fail to associate the engineer with the various improvements. The many industries of the country are brought before the public in their most advanced forms and as a result are benefited thereby.

Most people fail to realize that the engineering profession and the manufacturing industry are inseparable. The manufacturer is asking for technically trained men and it is for that reason that the faculties of Applied Science and Engineering are gaining such prominence in our universities to-day. The public admire and wonder at the X-rays, the wireless telegraphy, the wireless telephone, the aeroplane, the hydroplane, and hundreds of other products of discovery and invention, but are content to accept them as wonderful achievements without tracing them to their real source, the "engineering profession."

The only apparent reason that such is the case is that the profession does not receive the publicity which it merits. This is partially due, no doubt, to the unappreciative attitude of the daily press, but surely the engineer should take some step to remedy such a state of affairs. If a doctor of medicine discovers a new microbe or discovers a means to destroy one already known, the fact is printed, not in the medical journals alone, but is heralded in black type in the daily papers before the public as another achievement of the medical profession. On the other hand, an engineer may invent an appliance which will contribute immeasurably to the safety, comfort and convenience of the entire public, and as a rule, only those who read the engineering journals will be made acquainted with the fact.

The sooner the engineer realizes that he should receive more publicity, the sooner will the profession receive the recognition which it merits, in the minds of the public. The Exhibition has continued to expand rapidly and we trust that in the future every year may be an "Expansion Year." We also trust that the work of the engineer will be given proper publicity, as engineering work, and we feel sure that no better innovation or expansion could be instituted than the addition of an engineers' building to the group already in use.

Ernest W. Kay, '07, formerly office manager of the Canadian Westinghouse Co., Limited, Winnipeg, is now assistant manager of the Reinforced Brickwork Co., Limited, with head office at Winnipeg.

Wm. A. Duff, '01, formerly assistant bridge engineer for the Transcontinental Railway, at Ottawa, is now located at Moncton, N.B., as engineer of bridges for the Intercolonial Railway.

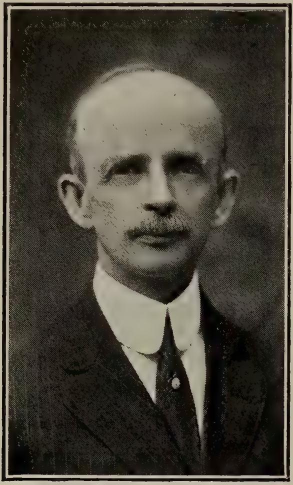
J. L. Stiver, '07, is with the Dominion Government Inland Revenue Department as inspector of gas and electricity, at Toronto.

H. O. Merriman, B.A. Sc., '10, is first assistant engineer and J. C. Nash, B.A. Sc., '10, is second assistant engineer in the Hamilton Hydro-Electric Department.

Geo. G. MacLennan, B.A. Sc., '10, is employed as engineer for the Foundation Co., Limited, on the construction of the C. P. R. Kildonan bridge in Winnipeg.

OBITUARY

Mr. David Burns, of the class of '83, died at his home in Pittsburgh, Pa., on Sunday morning, July 27th, 1913. Upon leaving the School, Mr. Burns spent a year on location and construction work on the Burlington and Missouri Railroad, in Nebraska, after which he became connected with the staff of the City of Toronto Waterworks. For three years, commencing in 1886, he held a fellowship in civil engineering at the School of Practical Science. He then became engaged in general survey work about the city. During 1891 and 1892 Mr. Burns was employed in the chief engineers' office of the Pennsylvania Railroad in connection with lines west of Pittsburgh. He resigned this position to enter the office of the city engineer of Allegheny, Pa., where he prepared plans for the elimination of grade crossings in that city.


Mr. D Burns, '83

After spending seven years on bridge construction, first in the Keystone Bridge Works and later with the American Bridge Co., he returned to railway work in 1902, with the West Side Belt Railroad.

In 1904 he became an instructor in the Carnegie Institute of Technology in Pittsburg, then known as the Carnegie Technical Schools. In April, 1905, his ability, derived from his experience in various branches of engineering, was recognized in his appointment as representative of the Carnegie Institute of Technology, in the erection of its buildings. From that time he occupied a chair among the members of the Faculty.

In 1890 Mr. Burns was granted a certificate as Ontario Land Surveyor. He was an associate member of the Canadian Society of Civil Engineers. In his demise the profession loses one of its grandest stalwarts. He had won the esteem of the engineering world at large, but more especially of the graduates of the School of Practical Science.

G. M. Cook, B.A. Sc., '13, is designing engineering for the Trussed Concrete Steel Co., Detroit. His addres is 224 Dougall avenue, Windsor, Ont.

E. V. Reid, '11, is with the firm Ross & MacDonald, architects, 1 Belmont street, Montreal, Canada.

Geo. S. Hodgins is assistant engineer of the mechanical department of the commission of the Transcontinental Railway, Ottawa, Ont.

DIRECTORY OF THE ALUMNI

This department began in November, 1912, and the entire list of graduates will be reviewed before the end of the year.

A number of corrections have been received bearing upon the part of the directory which has already been published, and our list of addresses is thus becoming more authoritative. If those whose names will appear during the next several months will assure themselves that our record of their location and employment is correct before publication, it will be improved still more, as there are some addresses upon our files that are long standing and hardly to be depended upon.

I (*Continued*)

Irvine, J., '89, deceased.

Irwin, H., '09, is managing editor of The Canadian Engineer, Toronto.

Irwin, W. J., '10, is with the Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

Isbister, J., '09, is electrical superintendent for the Mond Nickel Company at Victoria Mines, Ont.

J.

Jackes, F. B., '09, is commercial engineer for the Bell Telephone Co., Toronto.

Jackson, J. G., '03, was with the Toronto Hydro-Electric System as chief engineer until recently.

Jacken, F. C., '01, is a member of the firm of Jackson and Connelly, railway contractors, La Trique, Inc.

Jackson, W., '07, is with the Ontario Power Co., Niagara Falls, Ont., as field engineer on construction.

Jackson, C. B., '07, is in Kenilworth, Ill., in the estimating department, C. Everett Clark Co., Limited.

Jackson, J. E., '09, whose home is at Oxford Centre, Ont., is on survey work in the west.

James, O. S., '91, has a practice as assayer and chemical analyst, in Toronto.

James, D. D., '89, resides in this city. He is senior member of James and James, Ontario Land Surveyors.

James, E. A., '04, is chief engineer, York County Highway Commission, and member of the firm of James, London and Hertzberg, Toronto.

James, E. W., '09, resides in Winnipeg, Man., where he is engaged in the Public Works Department.

James, F. L., '10, whose home is in Tillsonburg, Ont., has no business address with this office.

Jarvis, R. H., '11, was in Regina, Sask., when last heard from.

Jepson, W. C., '06, is assistant engineer, Welland Canal office, Niagara Falls, Ont.

Jeffrey, D., '82, has had no record of address with us for a number of years.

Jermyn, P. V., '04, is engineer for the Reg. N. Boxer Wall Paper Co., Limited, of New Toronto, Ont.

Job, H. E., '94, is manager of the Toronto and Hamilton Electric Co., Hamilton.

Johnson, C. C., '09, whose home is in North Bay, Ont., is in the engineering office of Chipman and Power, Toronto.

Johnston, D. M., '02, resides in Toronto. He is with the Hydro-Electric Power Commission as Electrical engineer.

Johnston, H., '03, is city engineer of Berlin, Ont.

Johnston, H. C., '10, is looking after the Toronto interests of Anglin's Limited, building contractors, Montreal.

Johnston, A. C., '94, is vice-president and chief engineer of the J. M. Dodge Co., Philadelphia.

Johnston, S. M., '94, is engaged in land surveying in British Columbia and his address is Greenwood, B.C.

Johnston, H. A., '00, has Stettler, Alberta, for his present address. We do not know the nature of the work in which he is engaged.

Johnston, J. C., '00, has no address whatever with us at present.

Johnston, J. A., '00, resides at Ignace, Ont., where he is engaged in general contracting.

Johnston, C. K., '03, has a business as general merchant at Pefferlaw, Ont.

Johnston, R. H., '10, has a land

surveying practice at Edmonton, Alberta.

Johnston, W. J., '09, is a member of the firm of McKenzie, Broadfoot & Johnston, engineers and contractors, Vancouver, B.C.

Johnston, C., '06, is district engineer for the Canadian Northern Railway at Hanner, Ont.

Johnston, C. E., '09, is on Dominion land survey work in the West. His address is Fairford, Man.

Johnston, J. T., '08, is assistant engineer in the water power branch, Department of Interior, Ottawa.

Jones, J. E., '94, has for his address 149 Broadway, New York.

Jones, L. E., '11, is in the office of the surveyor-general, Victoria, B.C.

Jones, G. S., '05, has for his home address Smith Falls, Ont. He has no business address with us.

Jones, G. R., '06, is engaged in missionary work in China.

Jones, T., '06, is general manager of the Canadian Brake Shoe Company, Toronto.

Jupp, A. E., '06, is assistant engineer with Routly & Summers, Toronto and Haileybury, Ont.

K

Kay, E. W., '07, is assistant manager of the Reinforced Brickwork Co., Limited, with head office at Winnipeg.

Keefe, W. S. H., '04, is manager of the Light, Heat & Power Co., Fort Covington, N.Y.

Keele, J., '93, is engineer for the geological surveys branch, Department of the Interior, Ottawa.

Keffer, A. H. E., '09, is resident engineer, T. & N. O. Railway, near North Bay, Ont.

Keith, J. C., '10, is engaged in contracting and surveying, Calgary and Moose Jaw.

Keith, D. H., '07, is secretary-treasurer of Keith's, Limited, manufacturers and contractors, house and office fixtures.

Keith, H. P., '07, is a member of the city commissioner's staff, Moose Jaw.

Kelly, E. A., '11, is in Winnipeg in the construction department of the Canadian Pacific Railway.

Kemp, J. B. O., '09, is structural steel draughtsman for the Toronto Structural Steel Co., Toronto, Ont.

Kennedy, J. H., '82, is assistant chief engineer of construction for the Vancouver, Victoria & Eastern Railway & Navigation Co. at Vancouver, B.C.

Kennedy, H. G., '08, is at Cobalt, Ont., as engineer for the Cobalt Lake Mines.

Keppy, J. D., '06, is instructor and designer of tools for the Russell Motor Car Co., West Toronto, Ont.

Key, W. R., '09, is assistant engineer for the Turnbull Elevator Co., Toronto, Ont.

Keys, W. R., '08, is at Earlton, Ont., as resident engineer for the T. & N. O. Railway.

King, C. F., '97, is Toronto representative of the *Financial Times*, Montreal.

King, J. T., '10, is demonstrator in the department of mining, faculty of applied science, University of Toronto.

Kinghorn, A. A., '07, is general manager of the Asphaltic Concrete Co., 24 King St. E., Toronto.

Kingston, G. A., '10, Toronto representative for Jones & Glass Co., Montreal.

Kirkland, W. C., '84, is principal assistant engineer; drainage, sewage and water board, of New Orleans, La.

A "School" Pin ?
A Faculty Shield ?
A University Rug ?
Supply Department — Engineering Building.